A STUDY OF COVER FOR BOND IN THIN-SHELL PRECAST CONCRETE

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A STUDY OF COVER FOR BOND IN THIN-SHELL PRECAST CONCRETE

Submitted to
THE FACULTY OF RENSSELAER POLYTECHNIC INSTITUTE
in partial fulfillment of the requirements for
THE DEGREE OF MASTER OF CIVIL ENGINEERING

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A STUDY OF MINIMUM COVER FOR BOND IN THIN-SHELL PRECAST CONCRETE

SYNOPSIS

The purpose of the tests reported herein was to experimentally confirm the development of bond in thin-shell precast concrete.

Both pull-out and beam-type specimens were tested. Cover was the principal variable but bar types and diameters were also varied.

Pull-out tests indicated an increased developed bond with thicker covers and bond intensities of the magnitude of the American Concrete Institute working values. The beam tests indicated developed bond intensities of from three to five times the Code working values.

INTRODUCTION

The tests of the specimens reported herein constitute the second phase of an investigation of minimum bar spacing and protective cover in precast thin-shell concrete members. At the instigation of Mr. A. Amirikian, Chief of the Design Section of the Bureau of Yards and Docks, the first phase, consisting of tests on three bar pull-out specimens was done by Jubb, Loeffler, and Collins at Rensselaer Polytechnic Institute in 1948.

Numerous studies on bond have been made in recent years. These studies have produced a variety of conclusions; but each has produced new ideas on the theories and variables involved.

With the increasing use of thin-shell precast sections in many types of framing, conflicts arose between the present ACI and other building codes and the techniques peculiar to thin-shell concrete construction. Mr. Amirikian, in a recent paper presented to the ACI, stated in part, "The limitations of minimum bar spacing presently specified in ACI and other building codes when applied to precast concrete elements result in sections much larger than design and construction considerations would normally require. Unlike conventional work, where the use of some excess material in a framing does not appreciably affect its cost, any added weight or section in a precast framing is reflected in the form of an almost proportionate increase in cost. In some projects such additions may not only cause waste of materials but also may render them unprofitable undertakings. Obviously, the new technique of construction can ill-afford to carry such a needless burden during its critical period of development."

He also stated that, "It is frankly admitted that in the preparation of the ACI code no thought was given to the special conditions which now prevail in modern precast concrete construction. It is also true that there was no intention on the part of the compilers of the code that it should serve as a guide in all types of work. The new technique differs in many respects from the old." Therefore, in an effort to eliminate these differences in techniques and specifications, Mr. Amirikian has suggested, in part, that in setting up new specifications for thin-shell precast concrete construction: (a) ....."Reinforcing of slabs and secondary reinforcing in beams, girders, and columns shall be protected with concrete equal in thickness to one and one-half times the maximum size of the coarse aggregate, but in no case shall the thickness of covering be less than 3/8 inch." And (b) ....."The minimum clear distance between parallel bars shall be one and one-half times the maximum size of the coarse aggregate." ...... and to omit the one inch minimum spacing and cover requirement.

Jubb, Loeffler, and Collins investigated three-bar specimens varying bar size, aggregate, embedment, bar spacing, and cover. Their principal conclusions were: (1) bond resistance is proportional to cover and spacing for a given bar, and (2) bond stress is inversely related to bar diameter. Practically all of their specimens failed in tension, apparently due to a wedging action of the bar, rather than failing in bond. While their values for ultimate bond intensity compared favorably with the working values set forth in the ACI code, they did not provide a great enough safety factor to convincingly confirm the proposed changes in the code.

In continuing the investigation, the authors proposed to design and test two-bar specimens with a light truss between bars. Bar diameter, type bar, spacing, and cover were to be varied. The light truss was employed at the suggestion of Mr. Amirikian in an effort to eliminate tensile failures and to simulate diagonal reinforcement.

OUTLINE OF PROPOSED TESTS

VARIABLES IN SPECIMENS:

1. Clear Bar Spacing and Cover
    - A. 3/8" (1.5 x max. size aggregate)
    - B. 1/2" (2.0 x max. size aggregate)
    - C. 5/8" (2.5 x max. size aggregate)
2. Bar Types
    - A. Plain
    - B. Deformed
3. Bar Diameter
    - A. 3/4"
    - B. 1"

NOTES:

1. Aggregate Size = 1/4" maximum.
2. Embedment = 18".
3. Cover and bar spacing equal.
4. All specimens of two bars with a web truss.
5. All specimens poured horizontally.
6. Three specimens of each type cast.

TOTAL NUMBER OF SPECIMENS = 36

DESCRIPTION OF MATERIALS USED

SAND

The fine aggregate used throughout this investigation was a Long Island sand known locally as "Cow Bay" sand with a loose volume weight of 80-lb/cubic foot and an average moisture content of 2%

TABLE 1

SIEVE ANALYSIS OF SAND

U.S. Std. Sieve No.	Percentage Passing by Weight
4	100.00
10	91.80
20	34.10
30	9.24
40	1.40

CONCRETE

Le High Hi-Early and/or Atlas Hi-Early Portland cement which met all current ASTM standard specifications for type III cement were used for all castings. See Table 4 for all pour data.

COARSE AGGREGATE

The coarse aggregate employed was a local crushed limestone of a loose volume weight of 90-lb/cubic foot and an average moisture content of 0.4%.

TABLE 2

SIEVE ANALYSIS OF COARSE AGGREGATE

U.S. Std. Sieve No.	Percentage Passing by Weight
1/4"	100.00
4	66.02
10	2.99
20	0.29

REINFORCING BARS

Deformed bars were of the "Twin-twist" and "Bamboo" pattern of 3/4" and 1" nominal diameters. Plain bars were of 3/4" and 1" cold rolled stock. All bars had the normal amount of mill scale and little or no rust. The yield point and the ultimate strength were determined in the 100,000-lb. capacity Southwark-Emery testing machine.

TABLE 3

PHYSICAL PROPERTIES OF BARS

Nominal Bar Diam.	Type	Area $in.^2$	Yield Pt. psi	Ultimate St. psi
3/4"ø	D(T-t)	0.442	45300	83900
3/4"ø	D(B)	0.442	47500	74400
3/4"ø	PL	0.438	41500	65000
1"ø	D(B)	0.786	41300	69400
1"ø	PL	0.885	31700	49800

TABLE NO. 4

POUR DATA

POUR NO.	VOLUME RATIO	WEIGHT RATIO	W/C	ULT.COMP.ST.	DAYS CURED
1	1:1-1/2:1-3/4	1:1.6:2.1	6.8	3305	7
2	1:1-1/2:1-3/4	1:1.6:2.1	5.5	4000	12
3	1:1-1/2:1-1/2	1:1.6:1.8	5.5	5930	7
4	1:1-1/2:1-1/2	1:1.6:1.8	5.8	4997	8
5	1:1-1/2:1-1/2	1:1.6:1.8	5.8	4900	7
6	1:1-1/2:1-1/2	1:1.6:1.8	5.8	4420	4
7	1:1-1/2:1-1/2	1:1.6:1.8	5.8	5000	5

NOTE: (1) Zero slump in all pours except #1 which had a slump of 3".

(2) All test cylinders and specimens were cured in moist sand for the number of days specified in the last column.

(3) Computations were based on the following average values:
(a) Sand . . . . 80 PCF
(b) Coarse Aggregate . 90 PCF
(c) Cement . . . . . . 75 PCF
(d) % Moisture in Sand 2.0%
(e) % Moisture in Aggregate 0.4%

PREPARATIONS OF SPECIMENS FOR TESTING

After cutting the steel reinforcing bars to 39" lengths, two like bars were placed in a jig and four 1/4" square cross-pieces were arc welded onto the main reinforcing bars. These were placed at regular intervals over the 18" imbedment area with care being taken to have more than the tested cover at the ends of the specimen. They were placed at a 45° angle so as to form a light truss and to simulate diagonal reinforcing.

A wood end form and a 1" bearing plate with proper size holes and spacing were slipped over the free end of the main bars and then two 3" x 3/4" round lugs were arc welded to each main reinforcing bar at the loaded end and projecting perpendicular to the plane of the reinforcing bars. A cross-piece was then welded between lugs on opposite bars for greater rigidity and to prevent any tendency of the loaded ends to move with respect to each other during testing. All welding was done in the Welding Laboratory of Rensselaer Polytechnic Institute.

The bars were then ready for the casting of the specimens. All castings were made at the U.S. Naval Supply Depot, Scotia, New York. The wooden forms (see photographs) were oiled and properly spaced. The concrete was hand mixed in various proportions as shown in the table of data on the mixes. The concrete was poured horizontally in layers and rodded.

The forms were removed after one or two days and the specimens were then cured in moist sand as shown in the table with the data for the mixes.

Two test cylinders were poured at the time of each mix and cured in exactly the same way as were the specimens of that mix. The cylinders and the pull-out specimens were tested at the same time.

The bearing end of the concrete was capped for the four one-bar specimens but because the formwork left a smooth surface perpendicular to the axis of the reinforcing bars, no two-bar specimens required capping.

The specimens were then taken to the Strength of Materials Laboratory at Rensselaer Polytechnic Institute, placed in the testing apparatus, and pulled to failure.

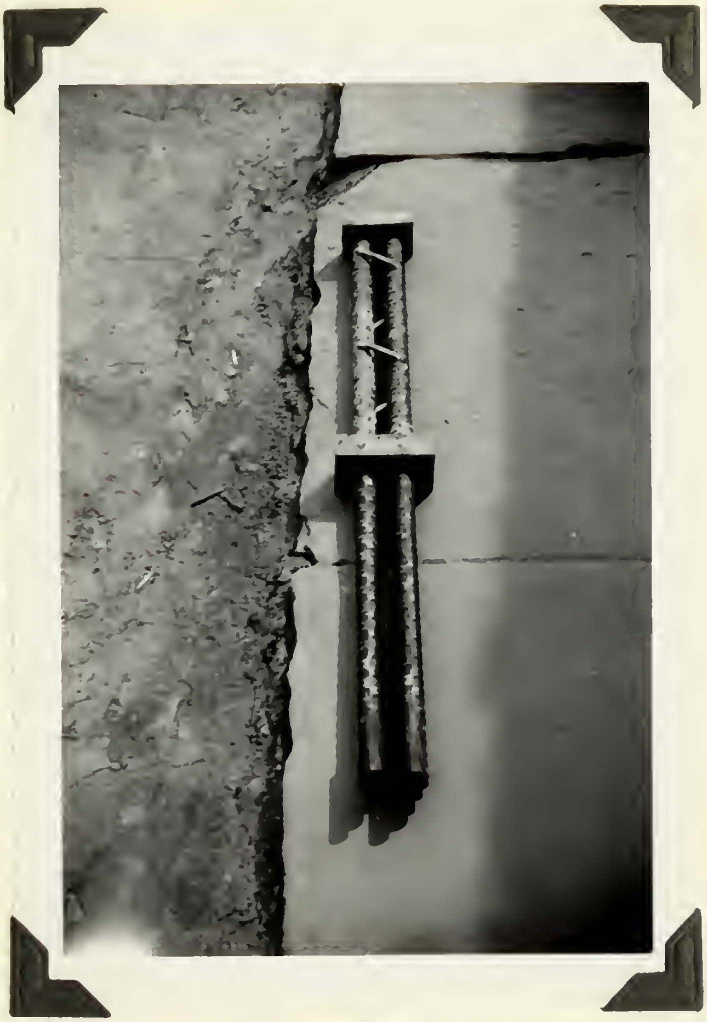


FIGURE #1

Typical steel assembly for a 1" deformed bar, 5/8" spacing and cover specimen. Note web truss, bearing plate, and lugs. End piece is wooden form used in casting.

FIGURE #2

Typical form assembly prior to casting pull-out specimens.

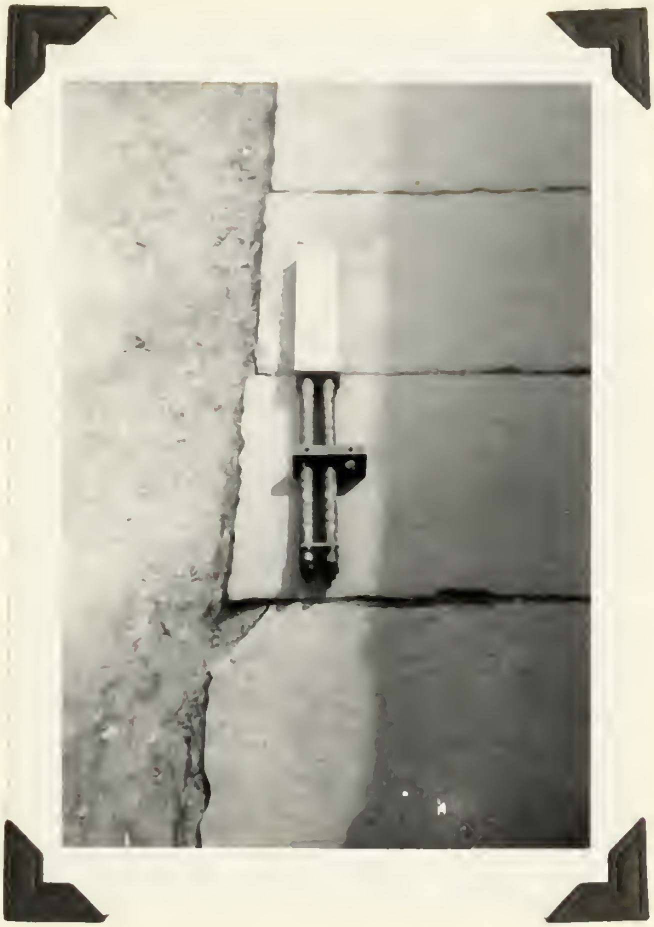
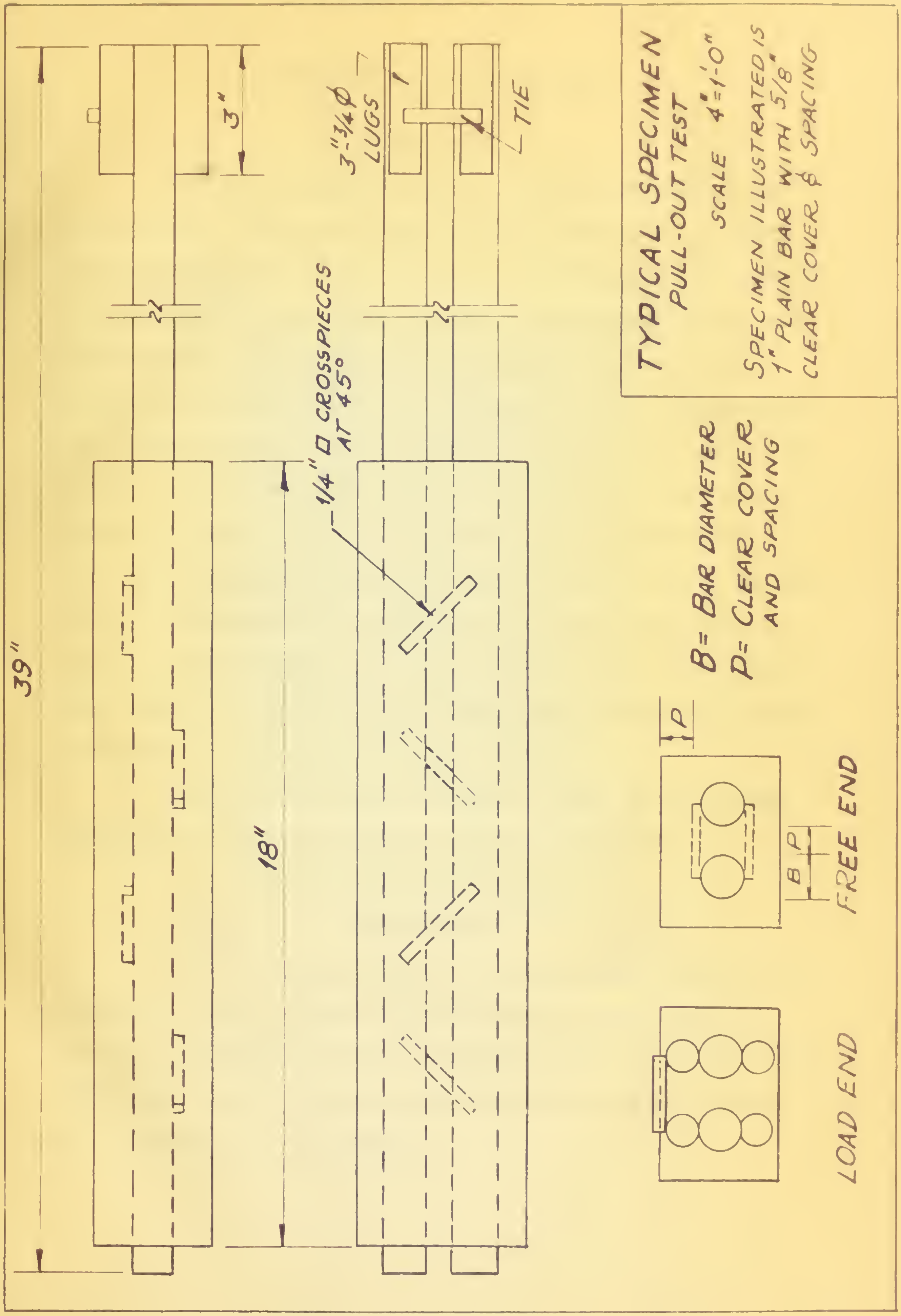


FIGURE ¢ 3

Completed pull-out specimen ready for testing. Note the tie welded across the lugs.

FIGURE 4



TYPICAL SPECIMEN
PULL-OUT TEST
SCALE 4"=1'-0"

SPECIMEN ILLUSTRATED IS 1" PLAIN BAR WITH 5/8" CLEAR COVER & SPACING

39"
3"
18"
3"-3/4 ϕ LUGS
TIE
1/4" □ CROSSPIECES AT 45°
B= BAR DIAMETER
D= CLEAR COVER AND SPACING
D
B
D
FREE END
LOAD END

DESCRIPTION OF TESTING APPARATUS

TESTING YOKES

The same testing yokes were used as were designed and employed by J.W. Collins, G.F. Jubb, and H.H. Loeffler, Jr., in their "Study of Minimum Bar Spacing for Bond in Thin-Shell Precast Concrete" (3 bar specimens) in the spring of 1948. This apparatus consists of two rectangular yokes similar in detail except that the vertical members of the upper yoke are longer (enough to test a specimen with a 24" embedment) than those of the lower yoke. All cross-pieces were of two 1" x 5" x 1'-6" cold rolled steel separated by the vertical members and a 1/4" spacer. The vertical members were two 5/8" x 4" bars. Each joint was pin connected with a 1-1/2" bolt. Each yoke was secured to its respective crosshead on the testing machine by a 2-1/2" x 1'-5" bolt, whose shank was machined on two sides to a thickness of 1-7/16" to permit the bolt to fit between the bars of the yoke cross-pieces.

The design load for the yokes was 100,000 lb. The design deflection for the cross-pieces was a practical minimum.

BEARING PLATES

The same bearing plates as designed by Jubb, Loeffler, and Collins were used in addition to several others of the same type which were fabricated for this investigation. These plates are 1" x 3" stock and are used to transmit the load from the lower cross-pieces of the upper yoke to the specimen.

FIGURE # 5

Pull-out specimen placed in testing rig prior to testing.

DESCRIPTION OF TESTING PROCEDURE

Considerable difficulty was experienced in leveling up the testing yokes in the machine. However, by (1) shimming the plate which supports the upper cross-piece of the upper yoke, (2) making sure the cross-pieces and vertical members were in the correct pairs, and (3) letting the upper cross-piece of the upper yoke hang free enough so that it was not bound by the upper crosshead of the testing machine, the yokes were level and produced no ascertainable eccentricity in loading.

The specimens, in the testing apparatus, were supported by a bearing plate which in turn was supported by the lower cross-piece of the upper yoke.

The load was transmitted to the specimen bars at the lower end from the upper cross-piece of the lower yoke through the bolt columns to the welded 3" lugs and thence to the main reinforcing bars.

The bolt columns consisted of four 5/8" x 1-1/4" tap screws, the head of each screw bearing on the welded lug and the nut bearing on the lower face of the upper cross-piece of the lower yoke. To have these bolt columns take the load, minimum clearance was used and all nuts were taken up finger tight. The first 1000 lb. of load was usually applied by hand.

The load was applied at approximately 5000 lb/min. using a 100,000 lb. capacity fluid support Southwark-Emery testing machine.

RESULTS FOR PULL-OUT SPECIMENS

TABLE NO. 5

SPECIMEN NUMBER	BAR DIAM.	SPAC.& COVER	TYPE BAR	POUR NO.	COMPRESS. STR. PSI	ULTIMATE	BOND INT.	RATIO TO f'_c	FACE COMP.	REMARKS
P-1	3/4	3/8	PL	1	3305	8000	94.3	.0285	----	P.TR. V.CR.
				(NOTE: P-1 and P-2 WERE NOT TRUSSED)						
P-2	3/4	3/8	D	1	3305	9600	113.0	.0345	----	same as P-1
P-3	3/4	3/8	PL	1	3305	12500	147.5	.0446	----	EX. COMP. V. CR. WF (1)
1	3/4	3/8	PL	2	4000	13500	159.2	.0398	3330	EX. COMP.
2	3/4	3/8	PL	2	4000	8000	94.2	.0235	1975	P.S.
3	3/4	3/8	PL	4	4997	16000	188.5	.0378	3950	V.CR.(4) BK.
4	3/4	1/2	PL	2	4000	18250	215.5	.0540	3520	V.CR.(3)
5	3/4	1/2	PL	2	4000	20400	241.0	.0604	3940	EX.COMP. V.CR.(4) BK.
6	3/4	1/2	PL	4	4997	20500	242.0	.0485	3960	V.CR.(4)
7,8,9	3/4	5/8	PL	NOT POURED						
10	3/4	3/8	D	2	4000	13400	158.0	.0395	3320	COMP.
11	3/4	3/8	D	4	4997	19900	234.0	.0469	4992	EX.COMP. V.CR. BK.

TABLE NO. 5 (CONT.)

SPECIMEN NUMBER	BAR DIAM.	SPAC.& COVER	TYPE BAR	POUR NO.	COMPRESS. STR. PSI	ULTIMATE	BOND INT.	RATIO TO f_c'	FACE COMP.	REMARKS
12	3/4	3/8	D	NOT POURED						
13	3/4	1/2	D	2	4000	11700	138.0	.0345	2260	P.S.COMP. V.CR.(2)BK.
14	3/4	1/2	D	4	4997	19750	233.0	.0467	3810	V.CR.(3)
15	3/4	1/2	D	NOT POURED						
16	3/4	5/8	D	2	4000	16500	194.5	.0486	2400	SP.COMP. V.CR.BK.
17	3/4	5/8	D	4	4997	23600	278.0	.0557	3440	V.CR.(3) BK.
18	3/4	5/8	D	NOT POURED						
19	1	3/8	PL	3	5930	9900	87.6	.0148	1775	P.S. SP. V.CR.(2)
20	1	3/8	PL	3	5930	12500	110.5	.0166	2240	V.CR.(4)BK. COMP.
21	1	3/8	PL	4	4997	19100	169.0	.0339	3420	COMP.
22	1	1/2	PL	3	5930	18300	162.0	.0273	2910	COMP. V.CR.(4)
22-A	1	1/2	PL	6	4420	21400	189.5	.0428	3420	V.CR.(2)
23	1	1/2	PL	3	5930	13200	117.0	.0198	2100	----
24	1	1/2	PL	3	5930	15400	136.0	.0229	2450	V.CR.WF(1)

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TABLE NO. 5 (CONT.)

SPECIMEN NUMBER	BAR DIAM.	SPAC.& COVER	TYPE BAR	POUR NO.	COMPRESS. STR. PSI	ULTIMATE	BOND INT.	RATIO TO f_c'	FACE COMP.	REMARKS
25	1	5/8	PL	5	4900	23500	208.0	.0424	2880	V.CR.(2)
26	1	5/8	PL	5	4900	27500	243.0	.0497	3360	V.CR.(3)
27	1	5/8	PL	NOT POURED						
28	1	3/8	D	5	4900	24900	195.5	.0399	3710	V.CR.(3)
29	1	3/8	D	5	4900	19900	156.0	.0319	2960	V.CR.(2)
30	1	3/8	D	NOT POURED						
31	1	1/2	D	5	4900	40100	314.0	.0642	5460	WF(1) EX. V.CR.(4)
32	1	1/2	D	5	4900	43800	344.0	.0703	5800	EX.COMP. V.CR.(short)
33	1	1/2	D	NOT POURED						
34	1	5/8	D	6	4420	46300	363.0	.0823	4820	WF(1&3); Crosspiece 2&4 bent; V.CR.(3)EX.
35	1	5/8	D	6	4420	48400	380.0	.0860	5040	EX.COMP. Bent crosspieces
36	1	5/8	D	NOT POURED						

TABLE NO. 5 (CONT.)

SPECIMEN NUMBER	BAR DIAM.	SPAC.& COVER	TYPE BAR	POUR NO.	COMPRESS. STR. PSI	ULTIMATE	BOND INT.	RATIO TO f_c'	FACE COMP.	REMARKS
THE FOLLOWING RESULTS ARE FOR ONE BAR SPECIMENS										
C-1	1	1/2	D	5	4900	7800	138.0	.0282	2420	V.CR. P.S.
C-2	1	1/2	D	6	4420	13200	233.0	.0528	4100	V.CR.(lower end only)
C-1-0	1(oiled)	1/2	D	5	4900	8500	150.0	.0306	2640	P.S.
C-2-0	1(oiled)	1/2	D	6	4420	12700	224.0	.0508	3820	V.CR.(lower end only)

KEY TO REMARKS

EX. ---- Explosive Failure

COMP. -- Compressive Failure

BK. ---- Buckling

V.CR.()- Vertical Cracks, number of faces indicated in ()

SP. ---- Spalling

P.S. --- Poor specimens (usually due to honeycombing)

WF() --- Weld Failure-Crosspieces numbered from bearing plate up. All trusses were arc-welded except P-3 which was brazed.

P.TR. -- Pull Through

FIGURE # 6

Pull-out specimen after loading showing a typical compression failure. Note cone at compression face near the bearing plate.

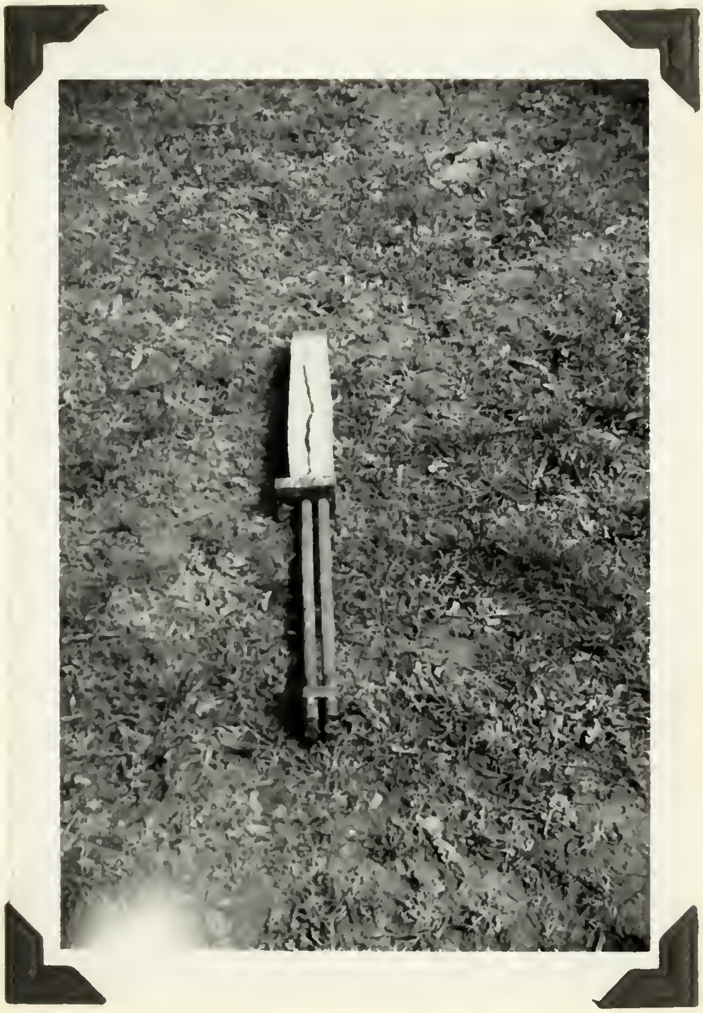


FIGURE #7

Pull-out specimen after loading showing typical pronounced vertical cracks between bars. (tensile failure).

FIGURE #8

Pull-out specimens after loading showing typical vertical cracks in the plane of the reinforcing bars. (tensile failure).

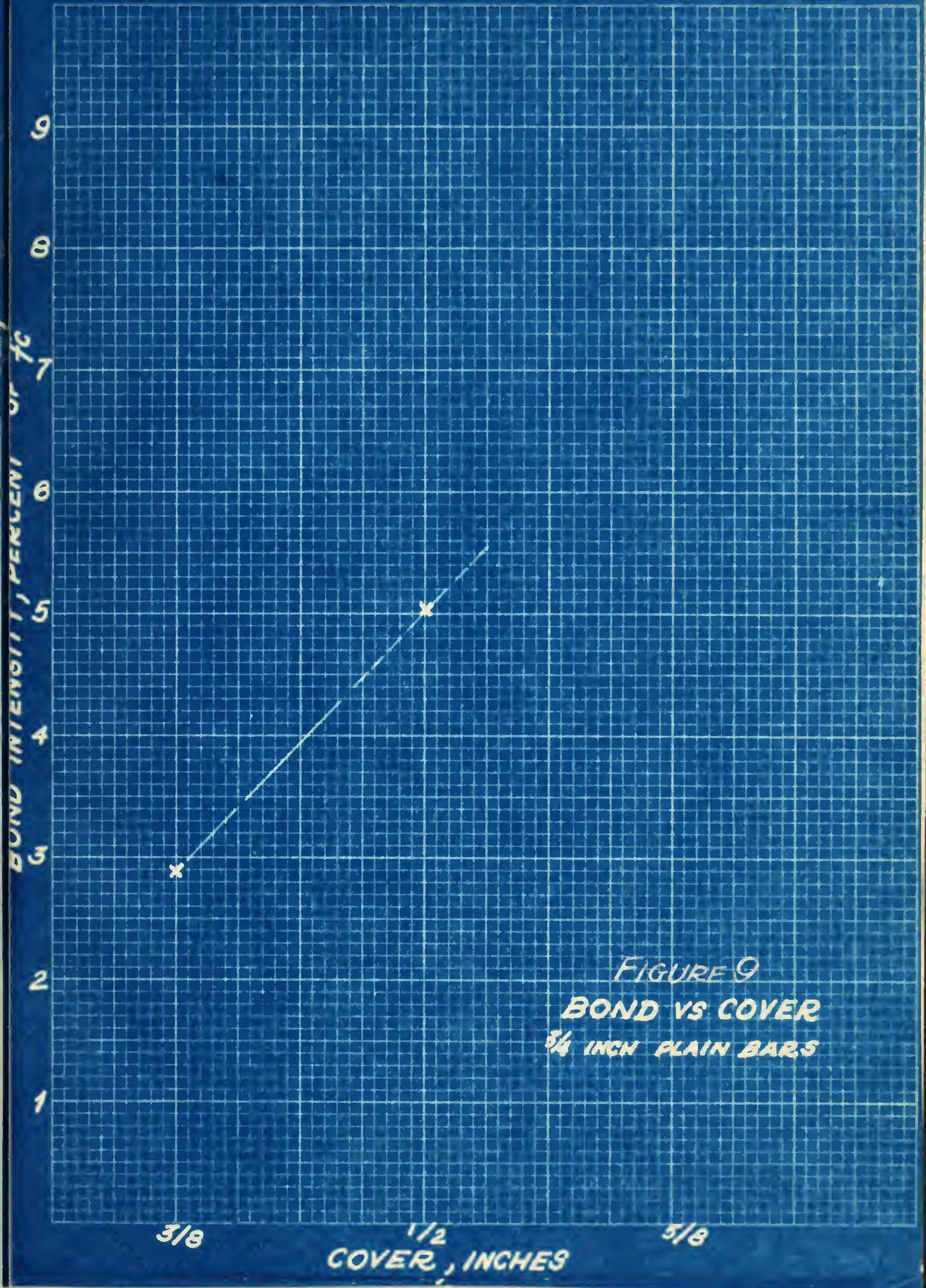


FIGURE 9
BOND VS COVER
3/4 INCH PLAIN BARS

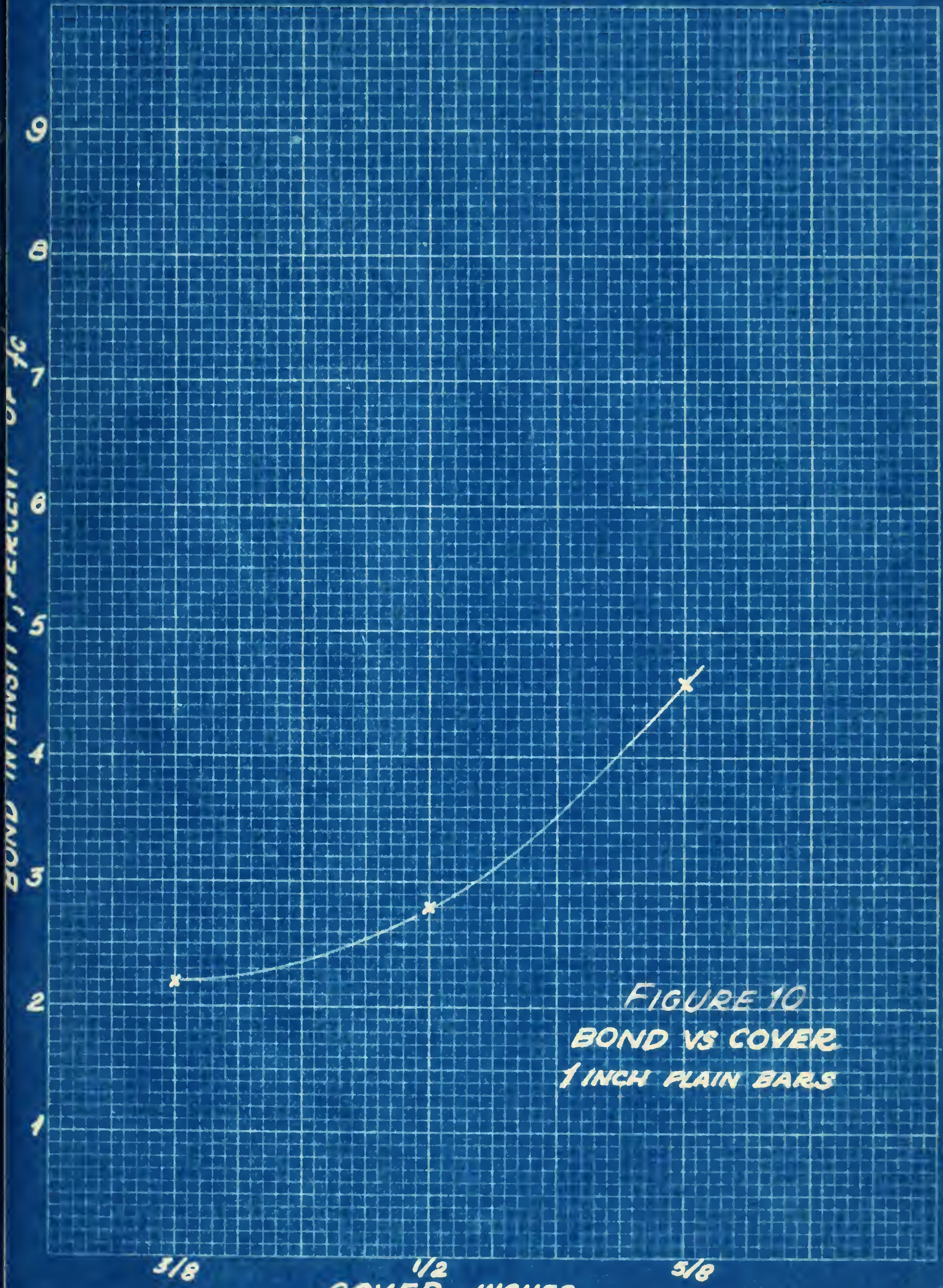


FIGURE 10
BOND VS COVER
1 INCH PLAIN BARS

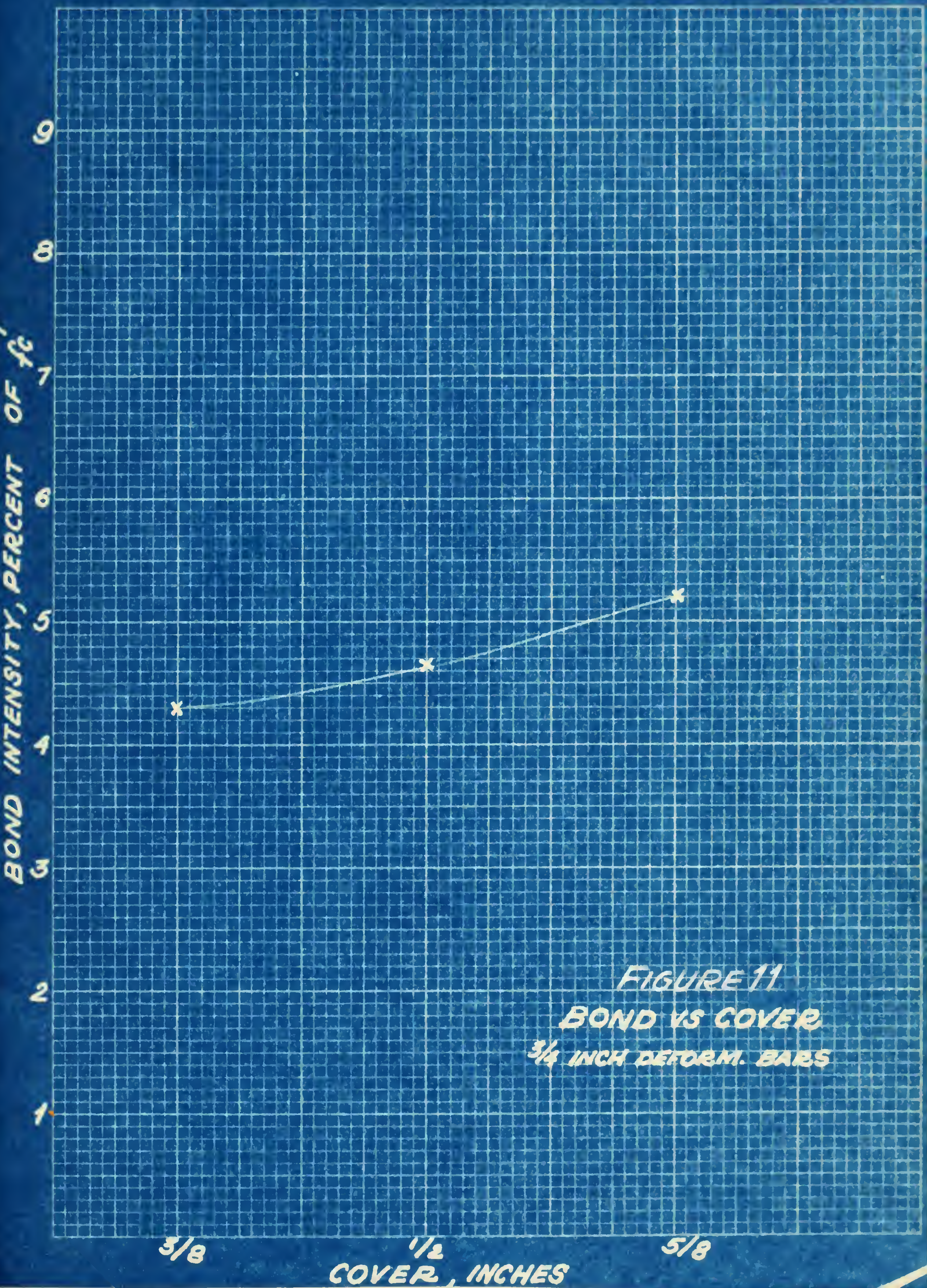


FIGURE 11
BOND VS COVER
3/4 INCH DEFORM. BARS

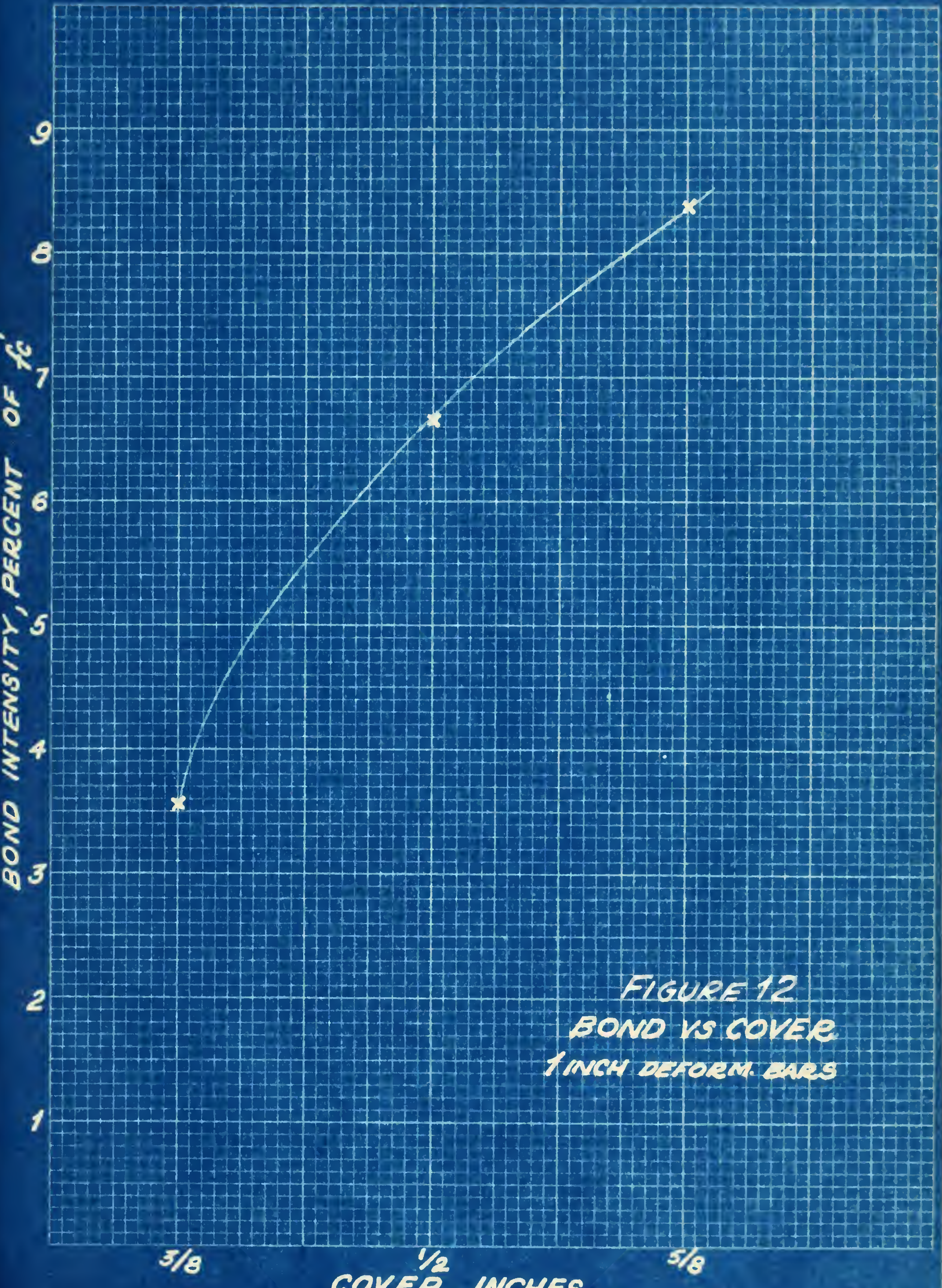


FIGURE 12
BOND VS COVER
1 INCH DEFORM. BARS

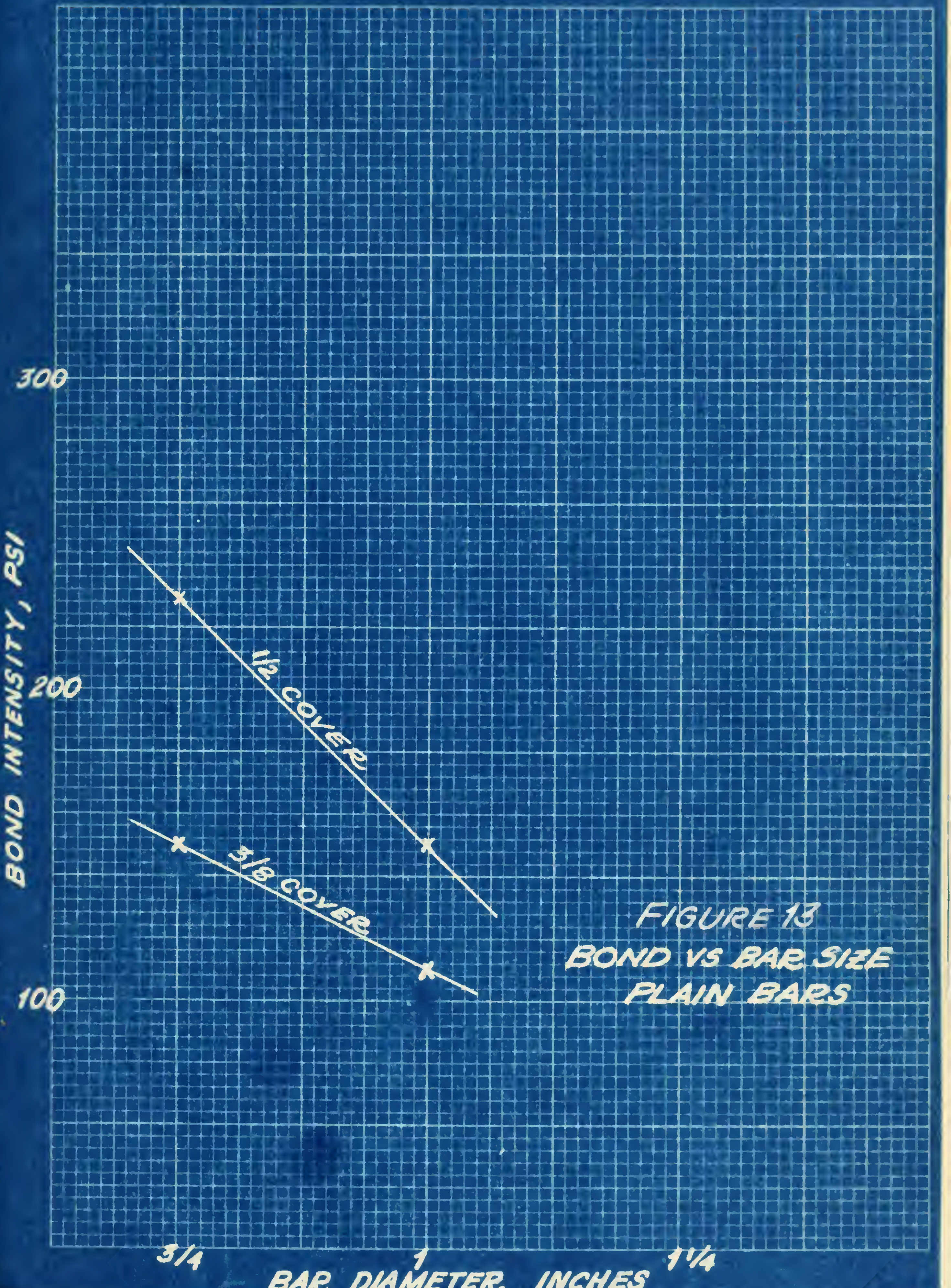


FIGURE 13
BOND VS BAR SIZE
PLAIN BARS

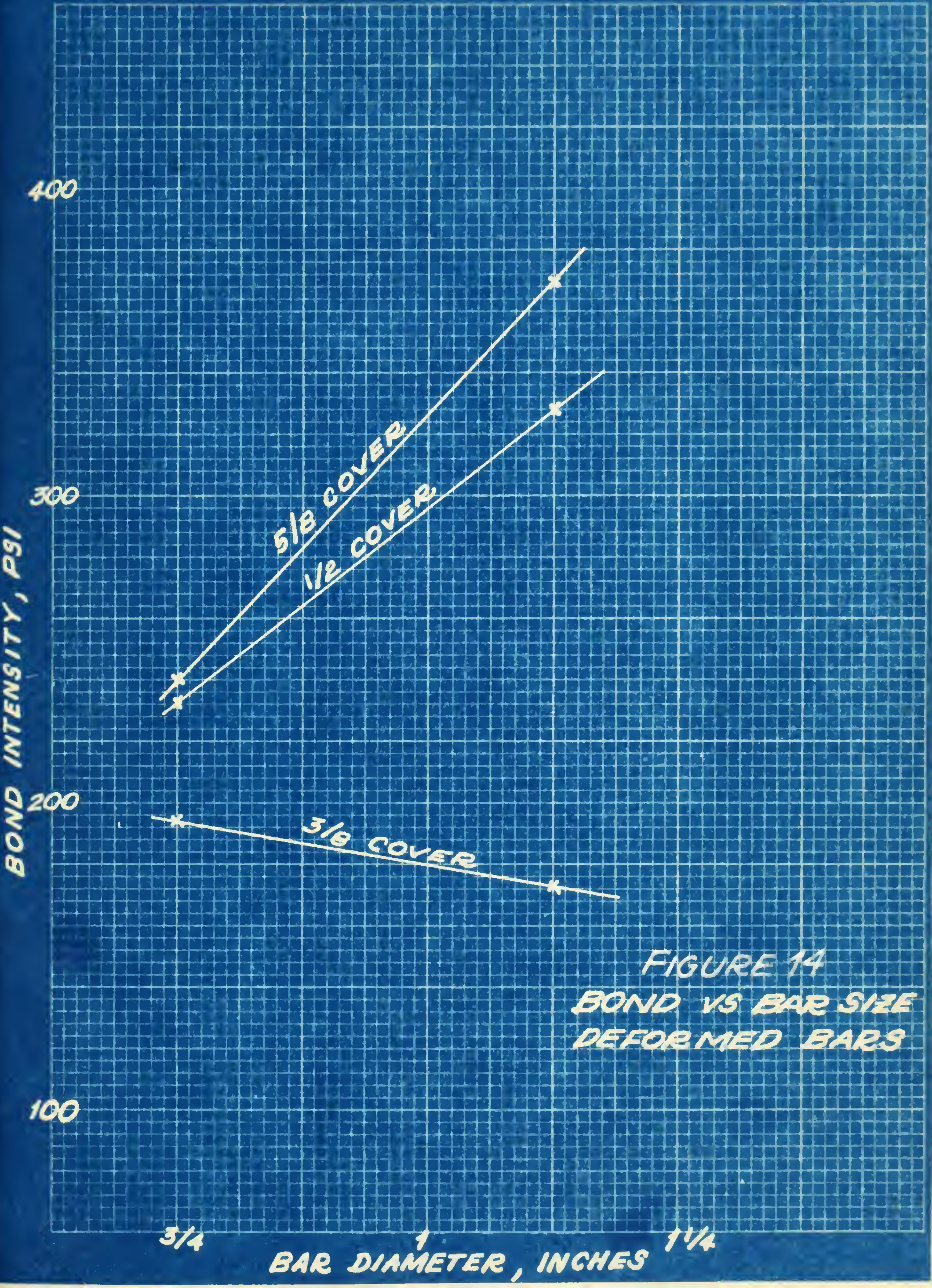


FIGURE 14
BOND VS BAR SIZE
DEFORMED BARS

TABLE NO. 8

REPRESENTATIVE RESULTS OF TWO-BAR SPECIMENS COMPARED TO THREE-BAR SPECIMENS (JUBB, LOEFFLER, COLLINS); SAME COVER, EMBEDMENT, ETC.

BAR DIAM.	SPAC.& COVER	TYPE BAR	BOND INTENSITY 2 BAR	BOND INTENSITY 3 BAR	RATIO TO f'_c 2 BAR	RATIO TO f'_c 3 BAR
3/4	1/2	PL	232.8	141.5	.0543	.0283
3/4	3/8	D	196.0	114.0	.0432	.0284
3/4	1/2	D	185.5	177.0	.0406	.0440
3/4	5/8	D	236.3	225.0	.0522	.0432
1	3/8	PL	122.7	66.5	.0224	.0133
1	1/2	PL	151.1	121.0	.0282	.0242
1	5/8	PL	225.5	161.0	.0431	.0322
1	3/8	D	175.8	85.0	.0339	.0212
1	1/2	D	329.0	146.0	.0673	.0363
1	5/8	D	371.5	212.0	.0842	.0538

NOTE: TWO-BAR SPECIMEN VALUES ARE AVERAGES OF SEVERAL LIKE SPECIMENS TESTED

THREE-BAR SPECIMEN VALUES ARE FOR THE ONLY COMPARATIVE SPECIMEN TESTED

DISCUSSION OF PULL-OUT TESTS

A total of 34 pull-out type specimens were cast and tested to failure. Of these, 27 were a part of the original schedule of 36 specimens, 3 were pilot specimens, and 4 were of a special type to be discussed later.

In general, failures were of the same types experienced by Jubb, Loeffler, and Collins. The most typical failure was by vertical tensile cracks in the plane of the bars, the presence of the web truss notwithstanding. (Figs. 7 and 8). Compressive failures of the bearing face were observed to some degree in many specimens as evidenced by spalling or a cone shaped wedge between the bars. (Fig. 6). Buckling was apparent in some specimens, probably resulting from secondary failures. Specimens which developed relatively high bond values failed explosively. (See results on specimens #31, 32, 34, and 35).

With respect to failures, it is noted that in some cases specimen dimensions limited the ultimate load to that of a compressive failure developing before the desired bond intensity could be realized. Nevertheless, failures, in general, occurred considerably before the ultimate compressive strength of the concrete was developed.

Bond as originally and strictly conceived was a phenomena of adhesion and friction. The concept has been enlarged to include deforming and special anchorage of the bars which, properly speaking, are processes of physically locking the reinforcing steel into the concrete mass. In an effort to obtain an indication of exactly what parts were played by friction and adhesion and by mechanical interlocking, four single bar specimens with 1/2" cover were fabricated.

The 1" deformed reinforcing bars were wire brushed to a uniform surface condition. Two bars were then oiled on the embedment surface to destroy adhesion and reduce friction. The other two bars were cast as brushed. It was expected that the oiled specimens would develop bond intensities lower by approximately the amount of adhesion and friction than the unoiled specimens. However, there was no such indication. Apparently the effect of pure bond is lost in the much higher apparent bond developed by the bar projections.

The results of the pull-out tests were disappointing. The use of the web truss had no appreciable effect in reducing the tensile failures. The anticipated increase in bond intensity over those values obtained by Jubb, Loeffler, and Collins was not nearly as high as hoped. (See Table 6). For plain bars and for any strength of concrete, the ACI Code gives an allowable bond stress of $0.04f_c'$ but not to exceed 160 psi. For plain bars in this investigation the results varied from approximately $0.039f_c'$ (170 psi) for 3/4" bars with 3/8" cover and spacing to $0.0436f_c'$ (226 psi) for 1" bars with 5/8" cover and spacing. For deformed bars and for any strength of concrete, the ACI Code gives an allowable bond stress of $0.05f_c'$ but not to exceed 200 psi. The deformed bars of this investigation showed results varying approximately from $0.0435f_c'$ (200 psi) for 3/4" bars with 3/8" cover and spacing to $0.0842f_c'$ (372 psi) for 1" bars with 5/8" cover and spacing. (See Tables 5 and 6 for exact and complete results). There was little evidence to substantiate the contention that code bond intensities could be used in thin-shell design with the minimum or very near minimum cover and spacing as utilized in this investigation.

There was a definite pattern of increased bond intensities as cover increased, confirming the findings on the three-bar specimens, and further dampening hopes that bond could be developed with thin covers. (See Figs. 9, 10, 11, and 12).

The effect of increased bar size on bond intensities was inconclusive. The results of the deformed bar analysis on this phase of the investigation are to be viewed with caution since the 3/4" and 1" bars were of different patterns. (Figs. 13 and 14).

If one definite final conclusion can be drawn from this investigation, it is that conventional pull-out tests are not the answer for thin-shell investigations. First, if bond intensities of satisfactorily high values were to be developed, the compression face would be loaded past the ultimate compressive stress. Second, the test prism assumes a long thin shape and is susceptible to column action. Any slight deviation from the perpendicular produces eccentric loadings, and localized failures. Third, the specimen because of its small size cannot be considered homogeneous. The averaging effect of a large concrete mass is lost, and localized failures instead of resulting in load distribution to the surrounding concrete, result in total failures. The small size of specimen also caused difficulties in pouring, the weight of concrete involved in a specimen being so slight as to require hand rodding of virtually every particle. Finally, the test is unrealistic from the standpoint of stresses. The concrete is in compression and the steel in tension in the pull-out tests whereas in the actual structural member, the stresses in both are of the same nature.

The 27 specimens previously mentioned covered the range of variables decided upon in the original schedule of tests, and, when

it became evident that the results were to be essentially negative, it was concluded that further tests of this nature would be of no practical value.

The results reduced to two alternative conclusions. Either the tests were right and the required bond could not be attained or the tests were in error and some defect in the testing procedure prevented development of the desired bond value. As previously stated, the testing procedure was viewed with increasing suspicion as the tests progressed.

With the approval of Admiral Combs, it was decided that a substantiating investigation utilizing beam type specimens would be undertaken in lieu of completing the schedule of pull-out tests. It was hoped that, by this means, the value of the pull-out results could be definitely established.

The following section will deal with the beam tests.

TEST OF BEAM-TYPE SPECIMENS

Design:

The specimens were designed in accordance with current specifications of the American Concrete Institute with two exceptions. First, working values for steel stresses were taken as 36000 psi, twice the working values prescribed by the American Institute of Steel Construction. This was done to avoid the use of two reinforcing bars throughout and a resultant decrease in bond intensity. The 36000 psi figure was well below the yield stress for the 3/4" deformed bars employed -- 47500 psi. Second, the design bond stress was $.15f'_c$, three times the permissible bond intensity allowed by the ACI. The latter was done to make the specimen particularly liable to failure in bond.

DESIGN FOR THIN-SHELL TEST BEAM USING 3/4" DEFORMED REINFORCING STEEL

Design Stresses -- A.C.I. Code

$f_c = .45f_c'$

$v = .06f_c'$ (web reinforced, no special anchorage)

$f_b = .25f_c'$

The beam is designed to develop a bond intensity of $u = .15f_c'$ three times the value permitted by the A.C.I. Code. The design is based upon the following assumptions:

$f_c' = 4000$ psi

$j = .85$

$K = 350$

$d = 10"$

$L = 36"$

To develop the required bond intensity

$V = u\sum_o jd$

$= (600)\ (2.356)\ (.85)\ (10) = 12000$ lbs

Required Area to resist Shear

$v = 240$ psi

$v = \frac{V}{bjd} \qquad bd = \frac{V}{vj} = \frac{12000}{(240)(.85)} = 58.8 \text{ in}^2$

Use a 6" x 10" section

Stirrup Design

$v' = 240 - 80 = 160$ psi

$S = 18"$

$b = 6"$

DESIGN FOR TEST BEAM (CONT.)

Stirrup Design (Cont.)

f_v = 18000 psi

f_c' = 4000 psi

x = 45°

B = 1.41

$\frac{\text{Max. } s}{d} = 1.0$ (Table)

Maximum Spacing = 10"

Try 1/2"∅ stirrups

$$\text{Max. } \frac{1}{s} = \frac{(\text{Max. } v')(b)}{(B)(A_v)(f_v)} = \frac{(160)(6)}{(1.41)(18000)(.20)} = .190$$

$$N = 6S(2 \text{ Max.} \frac{1}{s}) = 9(.380) = 3.42 \text{ or } 7 \text{ @ } 5"$$

Bearing Plates

End Areas

$$\frac{12000}{1000} = 12 \text{ in}^2$$

Use 1-1/2" x 8" Plate

Center Area

$$\frac{24000}{1000} = 24 \text{ in}^2$$

Use 6" x 4" plate

Check for Moments

Developed Moment = 14(12000) = 168000 ip

Max. Comp. Moment = Kbd^2

$= (350)(6)(10)^2 = 210000$ ip

DESIGN FOR TEST BEAM (CONT.)

Check for Moments (Cont.)

Max. Tensile Moment

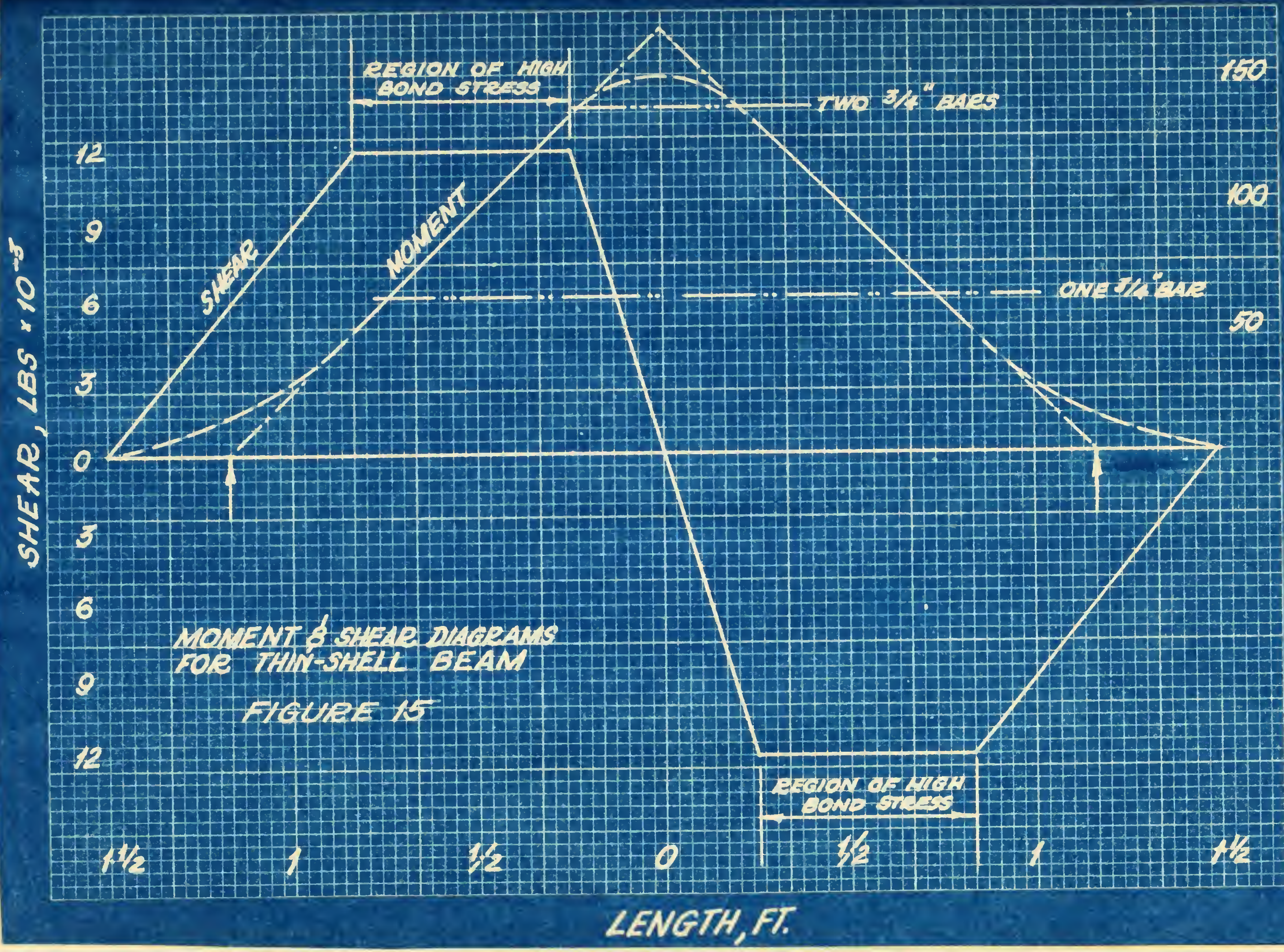
1 Bar $M = A_s f_s jd = (.44)(36000)(.85)(10)$

$= 134600$ ip

2 Bars $M = 269200$ ip

Wt. of Specimen

$$\frac{(36)(10)(6)(150)}{1728} = 187.5 \text{ lb}$$



MOMENT & SHEAR DIAGRAMS
FOR THIN-SHELL BEAM

FIGURE 15

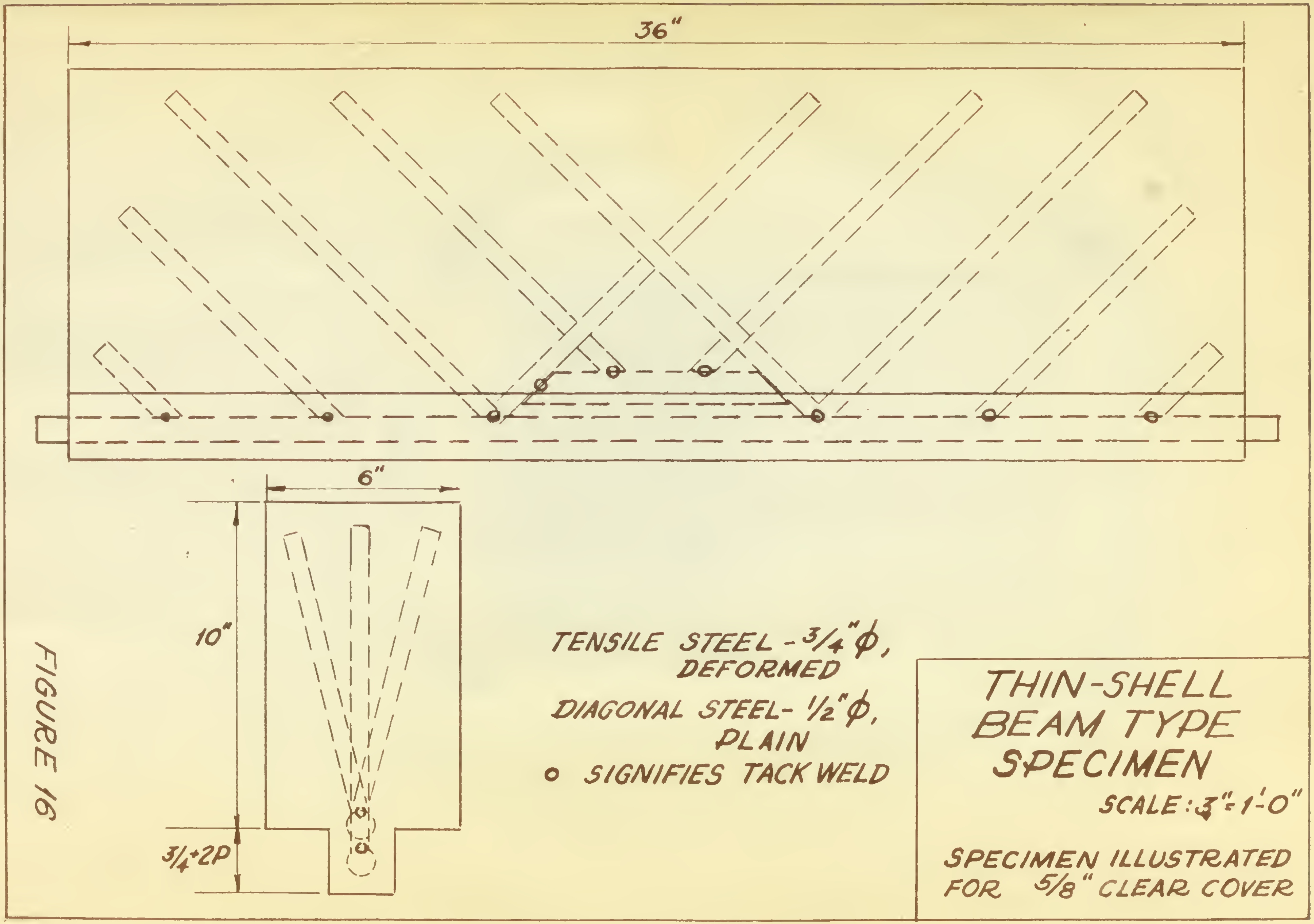


FIGURE 16

THIN-SHELL
BEAM TYPE
SPECIMEN
SCALE: 3" = 1'-0"

SPECIMEN ILLUSTRATED
FOR 5/8" CLEAR COVER

TENSILE STEEL - 3/4" ϕ, DEFORMED
DIAGONAL STEEL - 1/2" ϕ, PLAIN
○ SIGNIFIES TACK WELD

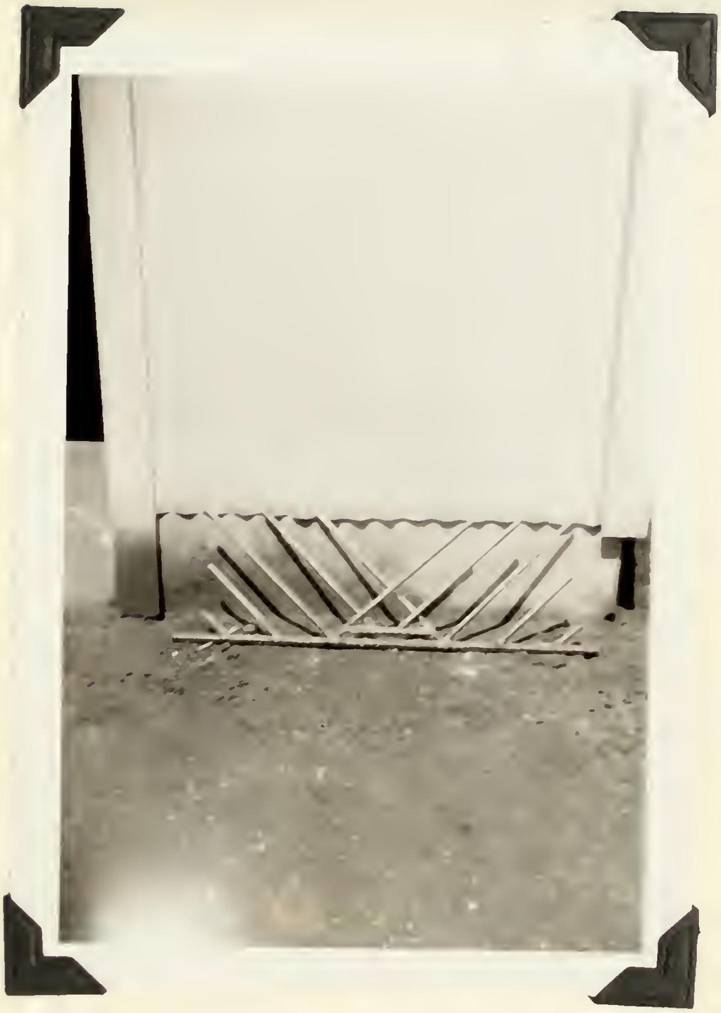


FIGURE #17

Steel assembly for a beam test specimen.

[illegible]

[illegible]

TEST OF BEAM-TYPE SPECIMENS (Cont.)

Fabrication:

In the construction of the beams, the diagonal steel was tack welded to the main tensile steel (Fig. 17). These welds were purposely made weak so as to barely withstand handling. This procedure was taken to minimize the interference of the diagonal steel with the tensile steel.

The completed steel assembly was bedded in the forms (Fig. 18) on a previously placed layer of concrete. Succeeding layers of concrete were placed and hand rodded until the pour was complete. The forms were stripped the next day and the beams were cured for six days in moist sand.

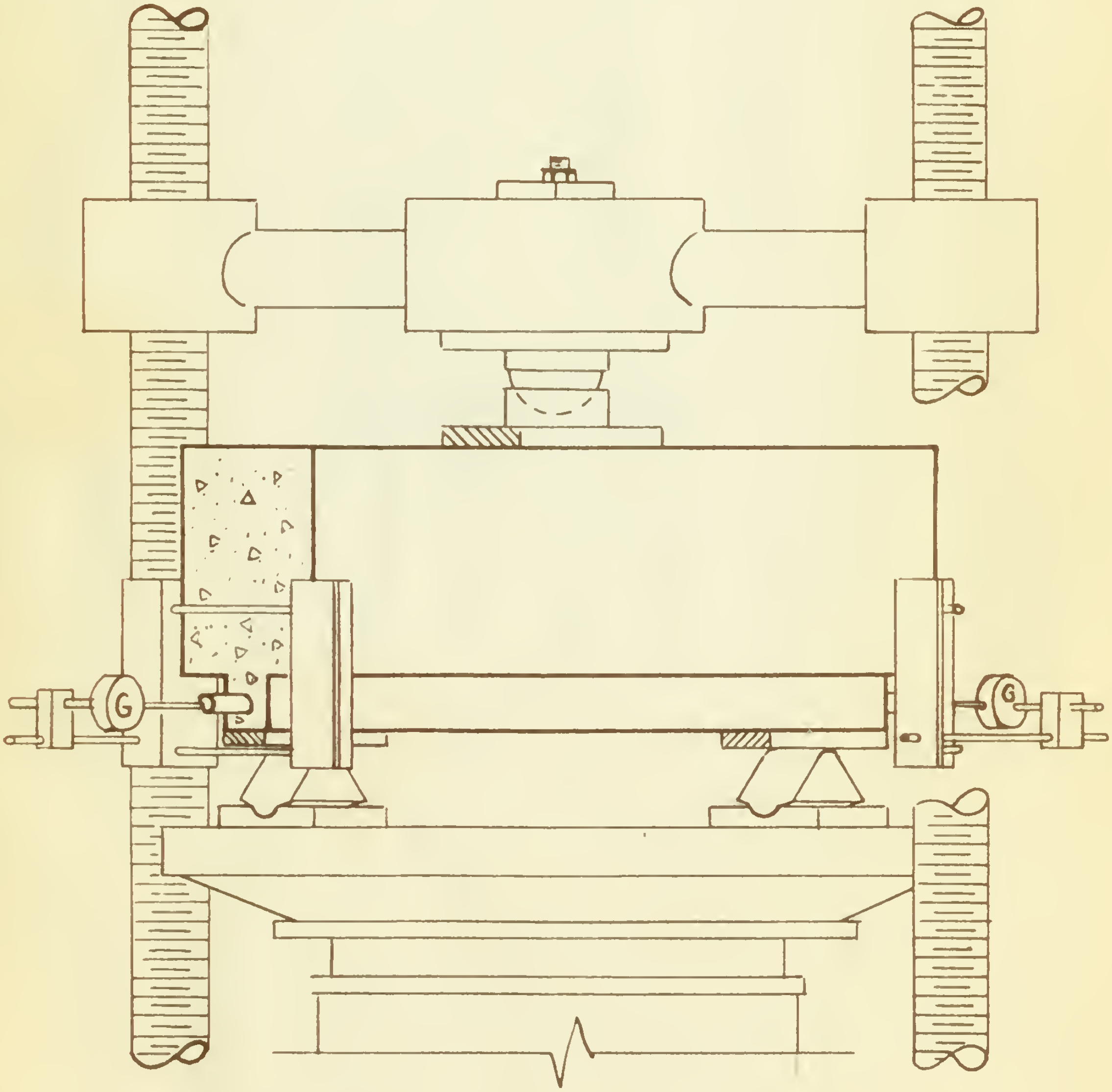
Testing:

The first step in the testing procedure was the capping of all compression faces with plaster of Paris. After the caps had set, the beam was placed in the Southwark-Emery testing machine (Fig. 19). The end bearing plates were mounted on knife edges, free to rotate about an axis parallel to that of the beam. The center bearing plate was loaded through a spherical bearing block. This minimized the possibility of eccentric loadings. Federal dial gages (0.001") were mounted bearing on the exposed ends of the reinforcing steel to measure slip between the steel and the mass of the concrete. The beams were then loaded to failure.

FIGURE #18

Formwork for beam test specimens. Note steel placed in left hand form ready for casting.

FIGURE 19



BEAM SPECIMEN, READY FOR TEST

RESULTS FOR BEAM TESTS

TABLE 7

RESULTS OF BEAM TESTS

Cover	First Slip Load	First Slip Bond	First Slip u/f'_c	Cracking Load	Cracking Bond	Cracking u/f'_c	Ultimate Load	Ultimate Bond	Ultimate u/f'_c	Type Failure
3/8"	24000	600	0.120	30000	750	0.150	54800	1370	0.274	Diagonal tension, Comp. end bearing
1/2"	26000	650	0.130	26000	650	0.130	52400	1310	0.262	Diagonal tension, Bond
5/8"	21000	525	0.105	22500	563	0.113	40800	1020	0.204	Comp. end bearing

Note: (1) Pour No. 7, $f'_c = 5000$ psi

TESTS FOR BEAM-TYPE SPECIMENS (Cont.)

Results and Discussion:

While the results of such a small number of specimens are not necessarily conclusive, the data obtained in these tests was very encouraging. The bond intensities, obtained from the conventional design formula, varied from $0.105f_c'$ to $0.274f_c'$ (Table 7). The only beam to exhibit typical bond failure, i.e. splitting in the plane of the bar, was the 1/2" cover specimen and this bond failure did not develop until virtually the ultimate load ($0.262f_c'$). There was no evidence of a bond failure in the 3/8" specimen ($0.274f_c'$). Finally, the 5/8" specimen, with the exception of a localized compression failure on an end bearing area, was perfectly sound after loading. The one hair-line crack which opened during loading could not be seen after removal of the load ($0.204f_c'$).

Beam tests appear to be a better approach to the determination of bond in thin-shell sections because of the elimination of many of the disadvantages of pull-out tests. (See "Discussion of Pull-Out Tests").

(1) The specimen can be designed for adequate bearing area to take load required for development of high bond intensities.

(2) Column action is eliminated.

(3) While the problem of localized failure is still present in the thin cover, the large mass of concrete in the compression area of the beam aids in compaction around the bar.

(4) The beams are flexural members as are the precast thin-shell sections used in actual practice.

These beam tests should be considered as a reconnaissance and, as such, are promising. The development of the desired bond intensities seems possible. However, considerable substantiating work with beam-type tests remains to be done before definite conclusions can be drawn. With an eye to such future work, the design, fabrication, and testing of the beams have been reported in somewhat greater detail than would otherwise be required.

CONCLUSIONS

1. Within the limits of the pull-out tests, bond resistance for a given bar size is directly related to the thickness of the surrounding cover.

2. Pull-out tests do not appear to satisfy the requirements for bond tests in thin-shell specimens.

3. Within the narrow limits of the beam tests reported herein, the development of sufficiently high bond intensities to assure an adequate safety factor with thin covers seems possible.

TOPICS FOR FURTHER INVESTIGATION WITH RESPECT TO BOND IN THIN-SHELL PRECAST CONCRETE

1. The determination of the effect of cover on bond by a comprehensive series of beam tests. This would require a beam design assuring a bond failure.

2. The possibility of increasing bond by bar treatment with a substance adhesive to steel and which will form an intimate mixture with the concrete adjacent to the bar.

3. An analysis of the load distribution on a reinforcing bar, using a section of steel tubing, with and without lugs, and fitted on the inside with resistance wire strain gages to determine the transfer of load from concrete to steel.

GENERAL REFERENCES

The following references contain information relative to bond in thin-shell precast construction. For detailed bibliographies covering the broad aspects of bond and tests for bond, the reader is referred to (2) and (3) below.

1. Amirikian, A., "Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members". Presented to the 46th annual convention of the American Concrete Institute, February, 1950. As yet unpublished.

2. Collins, J.W., Jubb, G.F., and Loeffler, H.H., Jr. "A Study of Minimum Bar Spacing for Bond in Thin-Shell Precast Concrete". Rensselaer Polytechnic Institute, June, 1948.
   This thesis covers the initial pull-out tests on three-bar specimens and laid the groundwork for the investigation which has been presented herein.

3. Gilkey, H. J., Chamberlin, S.J., and Beal, R.W. "Bond Between Concrete and Steel". Iowa Engineering Experiment Station Bulletin #147, 1940.
   This comprehensive report is the definitive work to date on bond investigations.

4. Amirikian, A., "Precast Concrete Structures", Journal of ACI, December, 1946.

5. Amirikian, A., "Precast Concrete Storehouses", Journal of ACI, June, 1947.

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